LIMIT STATE ANALYSIS OF REACTOR CONTAINMENT STRUCTURE – A PRELIMINARY STUDY

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dedicated

to

all hapless victims of

Chernobyl disaster

CERTIFICATE

This is to certify that the thesis entitled ''Limit State Analysis of Reactor Containment Structure - A Preliminary Study'' submitted by Abhai Kumar Rai in partial fulfilment of the requirements for the degree of Master of Technology of the Indian Institute of Technology, Kanpur, is a record of bonafide research work carried out under my supervision and has not been submitted elsewhere for a degree.

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Abhai

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LIST OF SYMBOLS

A_p	=	Sectional pressure of internal missile
A_{sm}	=	Area of cross-section of meridional reinforcement in the dome
A_x	=	Area of longitudinal reinforcement in the cylindrical shell per unit of length
A_θ	=	Area of circumferential reinforcement in the cylindrical shell per unit of length
a_{sm}	=	Cross-sectional area of each meridional reinforcement bar in the dome
a_{sc}	=	Cross-sectional area of each circumferential reinforcement bar in the dome
C^2	=	$4l^2/R\ t$
C/C_1	=	Compressive stress in concrete/steel in longitudinal direction in the cylindrical shell,
c	=	Compressive stress in concrete in the dome
c'/c_1'	=	Compressive stress in concrete/steel in circumferent direction in the cylindrical shell
D	=	Depth of penetration of internal missile
d	=	Diameter of internal missile
$d_c/d_n/d_t$	=	Distance of compression reinforcement/neutral axis/ tension reinforcement from the inner face
E	=	Total internal energy stored in the reinforcement
E_1/E_2	=	Internal energy in meridional/circumferential reinforcement in the dome
E_c/E_s	=	Modulus of elasticity of concrete/steel
K	=	Constant
l	=	Length of the cylindrical shell
M	=	Bending moment
M_x	=	Longitudinal moment per unit length of circumference
m	=	Reduced bending moment/modular ratio of concrete

m_x	=	Reduced longitudinal moment per unit length of circumference
m_p/m'_p	=	Reduced yield bending moment of reinforcement at outer/inner face
N	=	Axial force/missile shape factor
N'_θ	=	Circumferential stress resultant
$N'_{z\theta}$	=	Shear stress resultant
N'_z	=	Longitudinal stress resultant
n	=	Reduced axial force
n_p	=	Reduced yield axial force
$p/p(x)$	=	Internal pressure
$p_r/p_\theta/p_z$	=	Applied pressures in radial/circumferential/ longitudinal directions
p_c/p_t	=	Reinforcement percentage in compression/tension
R	=	Radius of cylindrical shell/dome
$T(^\circ C)$	=	Temperature difference across containment wall
t/t_c	=	Thickness of shell
t/t'	=	Tensile stress in steel reinforcement in longitudi cirfumferental direction in the cylindrical shell
t	=	Tensile stress in reinforcement in the dome
t_{sc}/t_{st}	=	Area of steel under compression/tension per unit width of shell
V	=	Shear force/velocity of internal missile
V	=	Increase in volume of dome
W	=	Work done by internal pressure/weight of internal missile
σ_r/σ'_r	=	Tensile/compressive yield stress of concrete
σ_y	=	Yield stress of reinforcement steel
μ_x/μ_θ	=	Reinforcement ratio in longitudinal/circumferenci direction
δ	=	Increase of radius of the dome
Δ	=	Extension of reinforcement bars

ABSTRACT

In this thesis safety aspects, of a typical reinforced concrete containment structure subjected to LOCA loads, have been studied by 'Limit State Analysis'. The approach used for collapse analysis is a combination of experimental and analytical techniques. Collapse modes experimently obtained by other investigators, have been used. Kinematic theorem has been used to obtain upper bound to collapse pressure for the dome. Besides uniform pressure the structure has been analysed for following loads:

(a) Asymmetric pressure caused by LOCA

(b) Thermal load (temperature gradient across the containment wall).

For the asymmetric pressure analysis the cylindrical shell section of the containment structure has been treated as a thin shell. Serviceability considerations studied are crack and penetration by internal missile.

CHAPTER - 1

INTRODUCTION

1.1 Nuclear Energy

Energy is the prime mover of economic growth and development. Naturally, for a better future society needs more energy for developmental activities and hence ample source thereof.

At the beginning of civilization, man's only source of energy was wood. Then, probably, came animal power. These two continued to be the major sources of energy for ages. Energy scene was revolutionized in the middle of 19th century when coal took over as the primary source of energy. Now oil seems to be replacing coal, and nuclear fuels seem all set to replace oil.

Industrialisation and attempts to improve quality of life of people have increased energy requirements manifold. The tremendous demand of energy will require diverse sources of energy. At present practical sources of energy are fossil fuels, hydro power, solar energy, wind, ocean tides, geothermal energy, and nuclear energy. The energy requirements of the world, in future, are expected to outstrip the economically recoverable fossil fuels. Another source of energy must then be developed to meet the energy demand. The contribution of wind, ocean, tides, solar, and geothermal energy are too limited and costly to make them alternative sources of energy

Hydro power too has some problems besides huge initial costs. This leaves nuclear fuels as the only economical sources for the future energy requirements. Though there are two processes by which energy can be generated from nuclear fuels-fission and controlled fusion, there are, still, major obstacles in the use of fusion process. This leaves us only with nuclear fission, at present.

Fission occurs when a fissionable nucleus captures a neutron. Capture upsets the internal force balance between neutrons and protons in the nucleus. The nucleus splits into two lighter nuclei, and an average of two or three neutrons is emitted. The resulting mass of products is less than that of the original nucleus plus neutron. The difference in masses appears as energy in an amount determined according to Einstein's formula $E = mc^2$, where E is energy, m is mass and c is the velocity of light. If one of the neutrons emitted is captured by another fissionable nucleus a second fission occurs similar to the first, another neutron may produce a third fission and so on. When the reaction becomes self-sustaining so that one fission triggers at least one more fission, the phenomenon is termed a chain reaction. The device in which this chain reaction is initiated, maintained and controlled is called a nuclear reactor.

There are several types of reactors currently in operation in different parts of the world. They can be

classified on the basis of type of fuel, coolant or moderator used or the energy of neutrons or a combination of these factors.

Though nuclear reactors are becoming more and more popular, questions are raised at times, regarding their safety aspects. It is natural considering the dangers associated with possible accidents at nuclear reactors.

1.2 Nuclear Reactor Safety

Chernobyl has given humanity what was only feared earlier. Yes; a major nuclear disaster leading to release of radioactivity in alarming amounts. This is not the place to discuss the causes or effects of the accident but it can be safely said that it has more than emphasized the importance of nuclear reactor safety and of reactor containment structure as a nuclear safety feature.

The basic design philosophy of nuclear power plants incorporates three levels of safety features. The first level of safety is to design the reactor and other components of the system to operate with a high degree of reliability. Also, all equipment should be able to operate with minimal chances of malfunctions. In order to achieve this, special emphasis is placed on the quality of materials used and workmanship. But careful design and construction does not necessarity eliminate all possibilities of incidents or malfunctions in the life of the reactor. The purpose of

the second level of safety is to provide means to forestall or cope with any such events. The reactor is consequently equipped with a protection system, the purpose of which is to safely accomodate a range of conceivable abnormal situations.

Still, the possibilities of some accidents occuring can not be completely ruled out. These accidents, called the design basis accidents, though highly unlikely, can be severe enough to endanger the wellbeing of the public. Hence, in order to provide additional measures to check and control accidents and, in case of failure in doing so, to protect the public from its hazards, engineering safety features are provided. Containment system is one of these features. Indeed, containment structure is the last safety measure. Even the thought, of what will happen if it fails, is horrible. Loss-Of-Coolant Accident (LOCA) is one of the design basis accidents (Type 8 accident as classified by WASH 1250) and can lead to release of radioactivity if not contained.

1.3 Loss-Of-Coolant Accident (LOCA)

LOCA refers to the discharge of coolant into the containment following the rupture of one of the lines leading to or from the core vessel. Loss-Of-Coolant Accidents fall into several categories depending on the size of the break in the primary coolant circuit. For the design basis accident,

however, a 'guillotine' (or double ended) break is postulated.

If such an accident takes place, the core will be heated up rapidly as there will not be sufficient coolant to cool it. In such eventuality, Emergency Core Cooling System (ECCS) comes into operation and neutronically reactor shuts down. But the residual heat has to be removed through other means. Naturally, LOCA leads to high pressure and temperature in the containment.

1.4 Containment Structure

If a large break should occur in the primary coolant circuit, the sudden decrease in pressure would cause almost all of the coolant initially in the reactor vessel to flash into steam. As a result of the coolant loss, some of the fuel rods could be damaged and radioactive fission products may be released. The containment system is the structure designed to contain essentially all the steam and radioactivity that would escape from the reactor vessel in a loss-of-coolant accident.

Containment structures used for different type of reactors (PWR or BWR) differ somewhat in shape, material used and other design considerations. In the beginning the BWRs were provided with steel containments while PWRs had concrete ones. But gradually concrete replaced steel as material of containment structure in all the reactors. The conventional concrete containment structure is a reinforced concrete structure

provided with a steel liner. Of late, prestressed concrete has been increasingly used for containment buildings all over the world. The choice between the two is indeed difficult. A large section of engineers and designers, these days, consider prestressed concrete to be more suitable for containment structure while another section feels that the conventional RCC containment structures are good enough and safe enough. The latest in this context is the growing use of double wall containment structure in which the inner wall in prestressed concrete is designed to resist internal pressure and prevent any leakage while the outer wall in RCC, is designed to resist external impacts (such as plane crashes etc.).

As far as shape is concerned, it generally conforms to the mechanical structure of the reactor. Most commonly it consists of a cylindrical shell capped by a hemispherical dome or a circular slab. And finally it must be mentioned that some reactors do not have any containment structure in real sense. But these are old ones and every new reactor is provided with a containment structure of one type or another.

1.5 Nature of LOCA Loads

Original design of the containment buildings assumed that applied LOCA pressures were uniform. This simplified assumption has been questioned by the U.S. Nuclear Regulatory Commission. Studies in this field have now shown that initially there is a asymmetric pressure distribution in the vessel which

becomes uniform after some time. Therefore there one two load conditions which the walls must resist during the LOCA:

1. Uniform internal pressure
2. Asymmetric internal pressure

Besides, there are certain other important considerations. One of them is the effect of temperature rise inside the containment following the LOCA. This leads to thermal stresses which should be within permissible limits. Sometimes the accident might be caused or accompanied by some small explosion in which case some portion of pipe or machinery might be thrown away as a missile and hit the wall. It is important to check the penetration of such missiles.

1.6 Scope of the Thesis

This work is an attempt to analyse a typical conventional RCC containment structure subjected to LOCA loads and find out whether it is adequately safe or not. For this purpose the containment structure of 600 MW Connecticut Yankee Atomic Power Plant has been chosen. The thesis consists of following parts:

(a) Collapse analysis of the containment structure subjected to uniform internal pressure.

(b) Analysis of the containment structure subjected to asymmetric internal pressure.

(c) Analysis of the containment structure subjected to thermal loads.

(d) Combined effect of asymmetric pressure and thermal loads.

(e) Assessment of damage by the internal missile.

These portions are contained in Chapters 3, 4 and 5. Chapter 2 is devoted to the review of relevant literature. Limitations of the work and suggestions for further work are discussed in the final chapter.

CHAPTER - 2

LITERATURE SURVEY

2.0 General

Earliest containment structures were designed as pressure vessels according to the provisions of 'Codes of Practice' on structural design in different countries. In most cases the design was conservative- essentially to take care of the element of uncertainty in assessment of loads and the risk involved. Earliest attempts at analysis were simplified ones, idealizing the structure and loads in certain aspects. Then, for some time, experimental studies were popular. Finally with the advent of better analysis techniques and computational facilities, much more rigorous and complete analyses were done using simulation and numerical procedures.

In this chapter the review of literature particularly relevant to the thesis work has been presented under different sections.

2.1 LOCA

The Lawrence Livermore National Laboratory in the U.S. did a study (funded by the U.S. Nuclear Regulatory Commission) under the name of Load Combination Program to, among other things, determine the probability of a large LOCA. Based on the results of the study, Chou [3] indicated that the probability of a double-ended guillotine break, either with or without earthquake is very small (of the order of 10^{-12}).

However, probability of a leak was found to be several orders of magnitude greater than that of a complete pipe rupture.

Thermal hydraulic analyses have been done to study the nature of LOCA loads. Based on such studies Eidinger and Manrique [4] mentioned that uniform pressure is not reached until 0.50 seconds after the pipe break, thus extremely high local pressures exist next to the ruptured pipe while normal ambient pressures exist 180° from the pipe break.

2.2 Thermal Effects

Ohsaki, Ibe and Aoyagi [5] experimently studied the effect of thermal loads on ultimate capacity of RCC cylindrical structures and found that thermal effects exert almost no influence on ultimate capacity of reinforced concrete structures.

Muto and others [6] studies effect of temperature on models of containment structures. They found that cracking and deformation of specimens due to thermal load is almost same, even if the amount of reinforcement is different. They also found that although thermal cracking causes decrease in rigidity ultimate earthquake resisting capacity is not affected by thermal loads.

In general creep has been neglected while calculating thermal stresses. It can be justified on the logic that since the accidental loads are short duration loads, creep is insignificant. Gol'denblat and Nikolaenko [12] gave a method, to calculate thermal stresses, which can easily take creep into account. In this method reinforced concrete is considered to

be a double-layer shell. One layer is metallic (steel) while the other is concrete.

Harada, Takeda, Yamane and Furumura [13] experimentaly studied the effect of elevated temperature on strength and elasticity of concrete. They found that these properties showed linear reductions. At 125°C the compressive strength, tensile strength and modulus of elasticity were 80 percent , 90 percent and 95 percent, respectively, of their original (unheated) values in case of normal portland cement concrete with sandstone aggregate. For other types of concrete the values were slightly different. The stress-strain curve also did not show much change when heated to temperatures less than 200°C.

2.3 Analysis for Internal Pressure

As already mentioned, there are two loads conditions which must be resisted during LOCA. These are asymmetric blowdown thrust and uniform pressure. According to Eidinger and Manrique [4], the asymmetric thrust part of this loading can be analysed statically with a Dynamic Load Factor of 1.15. Both load conditions cause similar steel reinforcement stresses near the break location.

Badrian and Iotti [7] tested some models of biological shield wall under LOCA loads and observed that displacements were negligible.

2.4 Shell Structures

The mechanical properties of a shell element describe its resistance to deformation in terms of separable streching and bending effects. Loads which are applied to the shell are carried, in general, by a combination of bending and streching actions which vary over the surface. One of the loading difficulties in the theory of shell structures is to find relatively simple way of describing the interaction between the two effects, Love [14] argued that for thin shell stretching rather than bending was the dominant effect. After that Lamb [15] and Basset [16] solved Love's general equation in the case of cylindrical shell and demonstrated the possibility of a relatively narrow boundary layer in which there was a rapid spatial transition between bending and stretching effect, with the width of the boundary layer being determined by the interaction between these effects.

Design of many thin shells is based on very approximate analyses. Billington [17] gave the reasons for this:
(1) rigorous analysis is so complex that the designer must necessarily resort to simplications, and (2) the load carrying capacity of thin curved structures often far exceeds the prediction of even the most refined available analysis.

There is no generally acceptable criterion to separate thin shells from thick shells. Different poeple have come out with different diameter to thickness (d/t) ratios depending

on the end conditions.

Billington [17] accepted the small-deflection theory as the basis for analysis of thin shell which implies that deflections under loads are small enough so that change in geometry of the shell will not alter the static equilibrium of the system.

Despite wide use of reinforced shells, their limit analysis is still in a relatively early stage of development. Massonnet and Save [18] believe that the best approach is to obtain limit loads by the work equation of the kinematic theorem from experimental collapse patterns with a sufficiently accurate yield surface. Sawczuk [19], Olszak and Sawczuk [20], and Haidukov [21] also advocated the approach to combine experimental and analytical techniques. Carmichael and Balakrishnan [18] found that analytical results vary considerably, depending on the yield criteria employed, both in load deformation response and in spread of inelastic and geometric nonlinearity characteristics.

Experimental investigations by Baker [22], Ernst and Marlette [23], Overtchkin [24] and Swczuk [19], strongly indicate that collapse mechanisms and limit loads do exist for reinforced concrete shells.

Drucker and Shield [25] developed procedures for finding upper and lower bounds on the limit pressure of symmetrically loaded thin shells of revolution by using the yield surface of a thin cylindrical shell as an approximation.

Hodge [26] obtained the upper and lower bounds on the yield point load of shells of uniform thickness by considering the related piecewise linear problems. The bounds can be determined if the yield stress in either tension or shear is known and are independent of the yield criterion.

Rozvany [27] proposed a theorem on the limit analysis of plates and shells and reported that the failure mechanism is associated with the lowest kinematically admissible multiplier or a moment field is associated with the highest statically admissible multiplier.

Haydl and Sherbourne [28], Khomyakov [37] and Guerlement and Lamblin [29] studied limit analysis of cylindrical shells under different loading conditions and using different methods. Haydl and Sherbourne showed that lower bound theorem is applicable in their analysis.

Szczepinski [30] presented a theory to determine the lower bounds on the bearing capacity of thin walled shell and plates loaded by continuously distributed bending and twisting moments.

Analytical approaches to the lower bound collapse loads under plane strain and plane stress rigid plastic deformations were developed by Inoue et al. [31]. The methods are applicable to a thick circular cylinders subjected to internal pressure and shear.

Kitching and Zarraki [1932] studied the lower bound limit pressures for a cylindrical shell with a milled rectangular slot simulating a part through crack. Open and close ended cylinders of different dimensions were analysed.

Considering cylindrical water tank and storage silos made of reinforced concrete as thin shell structure, Kumar [33] analysed those using lower bound theorem of limit analysis.

Limit analysis based on lower bound theory of reinforced concrete cylindrical shells like underground tank and silo were carried out by Adidam and Das [34] to determine the collapse load, mode of failure etc. for a given set of dimensional parameters and reinforcement.

Attempt were also made to study the failure mechanics of prestressed concrete containments. Murray , Simmonds and MacGregor [9] reported that ultimate failure mechanism is simple circumferential expansion of the cylinder wall leading to fracture of the post-tensioning tendons. Carretero and others [10] found that the structure ruins first by a loss of rigidity due to the cracking of the concrete so that the steel sustains the structure. The complete failure of the structure is obtained when prestressed steels go into plasticity.

Dubois, Avet-Flancart and Dulac [11] also gave the chronology of damage in the structure.

2.5 Damage by Internal Missiles

Investigators have come out with many empirical relations to calculate the depth of penetration by the internal missiles. Among the commonly used ones are Modified Petry Formula [35] used by U.S. Navy and Modified National Defence Research Committee Formula [36].

Finally it can be pointed out that a major drawback of almost all the analytical studies of containment structure has been the inadequate attention paid to thermal loads. In this thesis they have been given better treatment.

CHAPTER - 3

COLLAPSE ANALYSIS FOR UNIFORM INTERNAL PRESSURE

3.0 General

Safety of a structure against collapse is the first thing a designer is concerned with. It is of utmost importance to have adequate margin of safety against collapse. This chapter contains the plastic or collapse analysis of the containment structure performed to obtain the collapse pressure.

3.1 Structural Data and Other Information

Type of Reactor	:	Pressurised Light Water Reactor
Output	:	600 MW
Shape and Size	:	36.6 m high cylindrical shell of inner radius 20.6 m capped by a hemispherical dome of same inner radius.

Thickness of concrete cylindrical shell : 135 cm.

Thickness of concrete hemispherical dome: 76 cm

Grade of concrete : M 20 (f_{ck} = 20 MPa)

Design pressure : 40 psi (= 0.28 MPa)

Reinforcement : 50,000 psi (= 345 MPa) steel

Liner : ASTM A-408 steel

Thickness of the liner : 1/2'' in the dome

3/8'' in the cylinder

1/2'' at the bottom slab

Reinforcement details are shown in Figs.3.1 and 3.2.

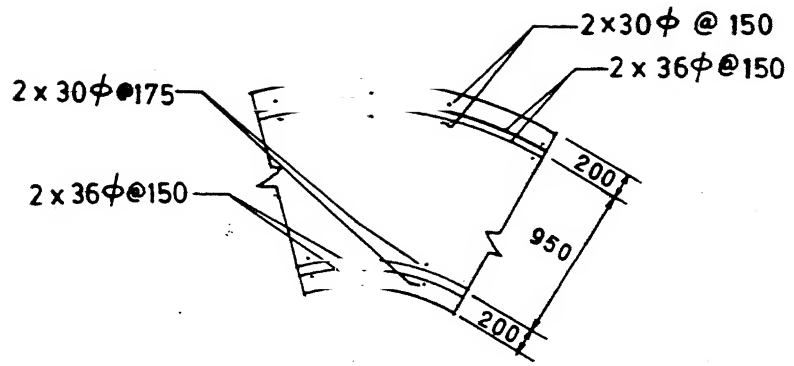

FIG. 3·1 REINFORCEMENT DETAILS OF THE CYLINDRICAL SHELL

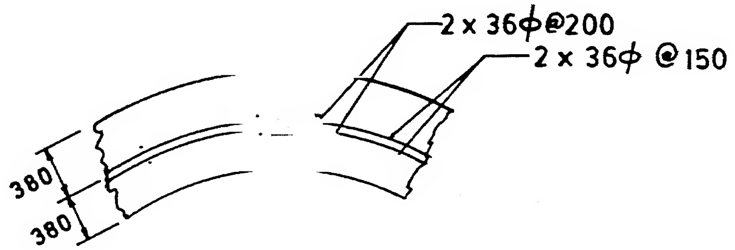

FIG. 3·2 REINFORCEMENT DETAILS OF THE HEMISPHERICAL DOME

3.2 Approach

The approach adopted for analysis is the one used and advocated by Massonnet and Save [18]. In this, limit loads are obtained by the work equation of the kinematic theorem. The kinematic theorem essentially gives an upper bound to the limit load. As the right collapse pattern is very difficult to obtain analytically, experimently obtained collapse pattern can be used. Thus the approach is essentially a combination of experimental and analytical techniques. In this thesis, however, collapse patterns obtained experimently by other investigators have been used for cylindrical shell and hemisperical dome.

3.3 Collapse Analysis of Cylindrical Shell

The collapse or plastic analysis of cylindrical shell portion of the containment structure can be discussed under following sections:

3.3.1 Yield condition

In this analysis the yield surface for reinforced-concrete cylindrical shells given by Sawczuk and Konig [39] has been used. This yield surface has been found suitable and is used by many investigators. In order to describe the yield condition generalized stresses are defined as

$$n = \frac{N}{\sigma_r' \, t}$$

$$m = \frac{M}{\sigma_r' \, t^2/4}$$

The reinforcement ratios

$$\mu_x = \frac{A_x}{2t}$$

$$\mu_\theta = \frac{A_\theta}{2t}$$

and the parameters

$$\alpha = \frac{\sigma_r}{\sigma'_r}$$

$$\beta = \frac{\sigma_r}{\sigma'_r}$$

Here N and M are axial force and bending moment, respectively, t is the thickness of the shell, A_x and A_θ are the areas of longitudinal and circumferential reinforcement per unit of length, and σ_r and σ'_r are tensile and compressive yield stresses of concrete. σ_y is the yield stress of reinforcement steel.

The yield surface is formed of two parabolic cylinders with equations

$$m_x(1+\alpha) + 2\,n_x^2 + 2\,n_x\,(2\beta\mu_x + \alpha - 1)$$
$$+ 2\beta^2\mu_x^2 - 4\beta\,\mu_x - 2\alpha = 0 \qquad (3.1)$$

and $$-m_x(1+\alpha) + 2n_x^2 + 2n_x(\alpha - 1) - 2\alpha = 0 \qquad (3.2)$$

limited by the planes with equations

$$n_\theta = 1 \qquad (3.3)$$

and $$n_\theta = -\alpha - \beta\mu_\theta \qquad (3.4)$$

The failure surface is shown in Fig. 3.5.

If the tensile strength of the concrete is to be neglected, α is taken to be zero. Also if axial loads are neglected, $n_x = 0$ and values of m_p, m'_p and n_p can be evaluated

from equations (3.1) and (3.2) by simply making $n_x = m_x = 0$ respectively. In the symbols m_p, m'_p and n_p p simply refers to the plastic or yield state.

The moment and membrane force capacities can vary from section to section longitudinally but the profile of the failure criterion remains essentially the same.

3.3.2 Collapse Mechanism

It has been concluded, on the basis of experimental studies on models of RCC containment structures like the one being analysed and other closed reinforced concrete cylinders subjected to uniform internal pressure, that the most common failure mode involves formation of hinge circles at the bottom, middle height (approximately), and top of the cylinder. In some cases the collapse pattern has been different but these can be attributed to the presence of local flaws in the material and significance of loads other than the uniform internal pressure (such as lateral loads). Using this information a collapse mechanism consisting of above mentioned hinge circle [Fig. 3.3] has been used in the analysis.

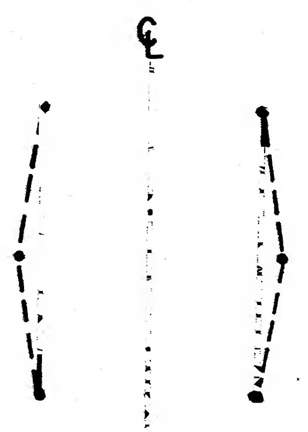

FIG. 3.3 ASSUMED COLLAPSE MECHANISM

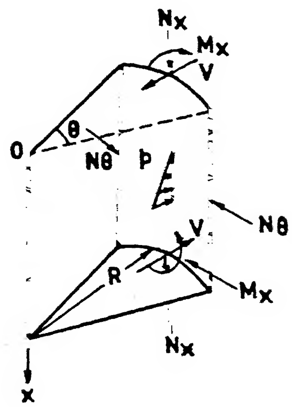

FIG. 3.4 TYPICAL SHELL ELEMENT

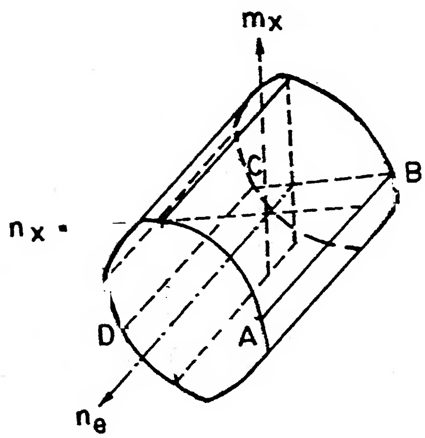

FIG. 3.5 YIELD SURFACE FOR REINFORCED CONCRETE CYLINDRICAL SHELLS

3.3.3 Equations of Equilibrium

As thermal effects do not seem to influence the ultimate capacity of RCC structures much [5] and dead loads are small the cylindrical shell has been considered to be subjected only to uniform internal pressure for the purpose of the plastic analysis.

Consider the shell element shown in Fig. 3.4. The equations of equilibrium of this shell element are:

$$R \frac{dV}{dx} - N_\theta - R\,p = 0 \qquad (3.5)$$

and

$$\frac{d\,M_x}{dx} = V \qquad (3.6)$$

where p is the uniform internal pressure, R is the radius of shell, V is the shear force and M_x and N_θ are longitudinal moment and hoop force, per unit length of circumference, respectively.

Eleminating V from these two equations

$$\frac{d^2 M}{dx^2} - \frac{N_\theta}{R} - p = 0 \qquad (3.7)$$

Defining a 'reduced pressure'

$$p' = \frac{p(x)\,R}{t\,\sigma_r'}$$

Equation [3.7] can be rewritten as

$$\frac{tR}{4}\,\frac{d^2 m_x}{dx^2} - n_\theta - p' = 0$$

or

$$\frac{l^2}{c^2}\,\frac{d^2 m_x}{dx^2} - n_\theta - p' = 0 \qquad (3.8)$$

where l is the length of the shell, $C^2 = 4l^2/Rt$, R is the radius of the median surface, and t is the effective thickness. The origin of the abscissa (x) is located at the top end of the shell.

For the assumed collapse mode, circumferential reinforcent at the hinge circles should fail in tension i.e. for this mechanism to occur $n_\theta = - n_p$.

Substituting this in equation (3.8) we get

$$\frac{l^2}{C^2} \frac{d^2 m}{dx^2} + n_p - p' = 0$$

or

$$\frac{d^2 m_x}{dx^2} = (p' - n_p) \frac{C^2}{l^2}$$

Integration of this equation gives

$$m_x = (p' - n_p) \frac{C^2}{2l^2} x^2 + Ax + B \qquad (3.9)$$

Boundary conditions:

$$m_x(0) = m_p = m_x(l) \qquad (3.10)\ \&\ (3.11)$$

Using these boundary conditions following values are obtained

$$B = m_p$$

and

$$A = - (p' - n_p) \frac{C^2}{2l}$$

Substituting these in eqn (3.9)

$$m_x = (p' - n_p) \frac{C^2 x^2}{2l^2} - (p' - n_p) \frac{C^2 x}{2l} + m_p$$

Conditions for failure

$$\left(\frac{d\, m_x}{dx}\right)_{x=l/2} = 0 \qquad (3.12)$$

and $m_x(l/2) = -m_p'$ (3.13)

Equation (3.13) implies

$$-m_p' = (p'-n_p)\frac{c^2l^2}{8l^2} - (p'-n_p)\frac{c^2l}{4l} + m_p$$

$$= -(p'-n_p)\frac{c^2}{8} + m_p$$

$$p' = \frac{8(m_p + m_p')}{c^2} + n_p$$

Since the structure has same amount (per unit length) of reinforcement on inner and outer faces, $m_p = m_p'$

Hence, $$p' = \frac{16\,m_p}{c^2} + n_p$$

3.3.4 Computations

Structural Data:

$$c^2 = \frac{4l^2}{Rt} = \frac{4\,(36.6)^2}{21.275\times1.35}$$

$$= 186.50$$

$$\alpha = \frac{\sigma_r}{\sigma_r'} = \frac{1}{20} = 0.05$$

$$\beta = \frac{\sigma_y}{\sigma_r'} = \frac{345}{20} = 17.25$$

$$\mu_x = \frac{0.017}{2\times1.35} = 0.0063$$

$$\mu_\theta = \frac{0.027}{2\times1.35} = 0.0050$$

Substituting these values in equations (3.1) and (3.2)

$$1.05\,m_x + 2n_x^2 - 1.4\,n_x - 0.508 = 0$$

and $$-1.05\,m_x + n_x^2 + 1.90\,n_x - 0.10 = 0$$

These are the equations of parabolic cylinders forming the yield surface.

By the time collapse occurs the concrete is already cracked. Hence it is appropriate to neglect tensile strength of concrete. For this case

$$\alpha = 0$$

Now the yield surface is given by following equations:

$$m_x + 2\, n_x^2 - 1.31\, n_x - 0.63 = 0 \qquad (3.14)$$

and

$$-m_x + 2\, n_x^2 - 2\, n_x = 0 \qquad (3.15)$$

Substituting $n_x = 0$ in (3.14)

$$m_p = 0.63$$

and substituting $m_x = 0$ in (3.15)

$$n_p = 1.0$$

Using these values p' is obtained

$$p' = \frac{16 \times 0.63}{186.5} + 1$$

$$= 1.054$$

Collapse pressure can now be calculated using this value

$$p = \frac{p'\, t\, \sigma_r'}{R} = \frac{1.054 \times 1.35 \times 20}{21.275}\ \text{MPa}$$

$$= 1.34\ \text{MPa}$$

Thus, Collapse pressure $= 1.34$ MPa

(= 4.78 x Design pressure)

3.4 Collapse Analysis of Hemispherical Dome

3.4.1 Collapse Mode and Yield Criterion

Reinforced concrete structures are believed to have collapsed when reinforcement fails. For the purpose of collapse analysis, the dome is treated as fixed at the joint with the cylindrical portion. Experimental studies of collapse of hemispherical domes subjected to internal pressure have led to the conclusion that hemispherical domes with fixed end condition take the shape of a semiellipsoid (with minor diameter $2b = 2R$ and major diameter $2a = 2(R+\delta)$) before collapse and internal energy is stored in reinforcement bars during extension. The collapse occurs when the bars yield by extension. Upper bound to collapse pressure can be obtained using kinematic theorem.

Kinematic or Upper Bound Theorem

This theorem, given by Prager [40], is stated as:

''In a rigid perfectly plastic continuum, plastic flow must occur under loads for which an unstable, kinematically admissible velocity field can be found''.

In other words, any load capacity obtained from a kinematically admissible displacement field is either correct or an upper bound to the correct load capacity.

3.4.2 Work Equation

The work equation can be obtained by equating external work done by internal pressure to the internal energy stored in

the reinforcement bars. Before doing so expressions have to be obtained for external work done and internal energy.

External work done (by internal pressure)

= internal pressure x change in volume

i.e.
$$W = p\,[\Delta V]$$
$$= p\,[\tfrac{2}{3}\pi R\,(R+\delta)^2 - \tfrac{2}{3}\pi R^3]$$
$$= \tfrac{2}{3}\pi R^3 p\,[(1+\tfrac{\delta}{R})^2 - 1]$$
$$= \tfrac{2}{3}\pi R^3 p\,[1 + 2\tfrac{\delta}{R} + \tfrac{\delta^2}{R^2} - 1]$$

neglecting second order term δ^2/R^2

$$W = \tfrac{2}{3}\pi R^3 p\,\frac{2\delta}{R}$$

or
$$W = \tfrac{4}{3}\pi R^3 \delta p \qquad (3.16)$$

Internal Energy: As already mentioned internal energy is stored in the meridional and circumferential reinforcement in the process of extension. Hence

$$E = \Sigma A_{sm}\,\sigma_y\,\Delta$$

where Δ is the extension of the bar in the plastic region.

(a) Meridional Reinforcement

Original length of reinforcement bars

$$l' = \pi R$$

Length of transformed or extended bars

s = Arc length of the semiellipse

i.e.
$$s = 2a\int_0^{\pi/2}\sqrt{1-e^2\,\mathrm{Sin}^2 x}\;dx$$

where x is measured with the minor axis and

$$e^2 = 1 - \frac{b^2}{a^2} = \frac{a^2-b^2}{a^2}$$

$$= \frac{(R+\delta)^2 - R^2}{(R+\delta)^2}$$

$$= \frac{R^2 + \delta^2 + 2R\delta - R^2}{(R+\delta)^2}$$

$$= \frac{2\delta}{R} \text{ (approximately)}$$

Expanding the function in the integral using binomial expansion,

$$s = 2a \int_0^{\pi/2} \left(1 - \frac{1}{2} e^2 \sin^2 x - \frac{1}{2} \cdot \frac{1}{4} e^4 \sin^4 x - \frac{1}{2} \cdot \frac{1}{6} \cdot \frac{1}{6} e^6 \sin^6 x - \ldots\ldots\right) dx$$

if eccentricity is small, s can be approximated by following expression [38]

$$s = \pi a \left(1 - \frac{e^2}{2^2} - \frac{1^2 \times 3}{2^2 \times 4^2} e^4\right)$$

$$= \pi (R+\delta) \left(1 - \frac{e^2}{4} - \frac{3}{64} e^4\right)$$

Now extension of one reinforcement bar

$$\Delta = s - l'$$

$$= \pi (R+\delta) \left(1 - \frac{e^2}{4} - \frac{3}{64} e^4\right) - \pi R$$

$$= \pi \left[\delta \left(1 - \frac{e^2}{4} - \frac{3}{64} e^4\right) - R \left(\frac{e^2}{4} + \frac{3 e^4}{64}\right)\right]$$

Substituting $e^2 = \frac{2\delta}{R}$

$$\Delta = \pi \left[\delta \left(1 - \frac{\delta}{2R} - \frac{3}{16} \frac{\delta^2}{R^2}\right) - R \left(\frac{\delta}{2R} + \frac{3}{16} \frac{\delta^2}{R^2}\right)\right]$$

$$= \pi \left[\left(\delta - \frac{\delta^2}{2R} - \frac{3}{16} \frac{\delta^3}{R^2}\right) - \left(\frac{\delta}{2} + \frac{3}{16} \frac{\delta^2}{R}\right)\right]$$

neglecting δ^2 and other higher order terms as δ is small,

$$\Delta = \frac{1}{2} \pi \delta$$

$$E_1 = n_m a_{sm} \sigma_y \Delta$$

$$= \frac{1}{2} \pi n_m \sigma_y a_{sm} \delta \quad (3.17)$$

where n_m is the number of meridional reinforcement bars and a_{sm} is the area of cross-section of each bar.

(b) Circumferential Reinforcement

Consider any point (h,k) on the circular section through the middle of the hemispherical dome. The coordinates can be converted related to spherical coordinates by following expressions:

$$h = R \cos \theta$$

and $$k = R \sin \theta$$

where R is the radius of the circular section and θ is measured from the ordinate.

Consider the circumferential reinforcement bar passing through the point (h,k).

Length of the bar $= 2\pi h$

$= 2\pi R \cos \theta$

After the hemisphere has taken the shape of a semiellipsoid, any section through the middle will be an ellipse. Any point on this ellipse is given by

$$h = R_e \cos \theta \text{ and } k = R_s \sin \theta$$

where $R_\theta = \sqrt{(R+\delta)^2 \sin^2\theta + R^2 \cos^2\theta}$

Increase in length of reinforcement bar

$$L' = 2\pi \cos\theta \left[\sqrt{(R+\delta)^2 \sin^2\theta + R^2\cos^2\theta} - R\right]$$

$$= 2\pi \cos\theta \left[R\left(\left(1+\frac{\delta}{R}\right)^2 \sin^2\theta + \cos^2\theta\right)^{1/2} - R\right]$$

$$= 2\pi \cos\theta\left[R\left(\left(1 + 2\frac{\delta}{R} + \frac{\delta^2}{2R^2}\right)\sin^2\theta + \cos^2\theta\right) - R\right]$$

neglecting second order term $\frac{\delta^2}{R^2}$ and simplifying

$$L' = 2\pi \cos\theta\left[R\left(1 + \frac{2\delta}{R}\sin^2\theta\right)^{1/2} - R\right]$$

$$= 2\pi \cos\theta\left[R\left(1 + \frac{1}{2}\frac{2\delta}{R}\sin^2\theta + \ldots\right) - R\right]$$

again neglecting higher order terms

$$L' = 2\pi \cos\theta\,(\delta \sin^2\theta)$$

$$= 2\pi\ \delta \cos\theta \sin^2\theta \qquad (3.18)$$

Since the reinforcement bars are at different θ, the extension will have to be calculated for different bars and summed. Hence

$$E_2 = \Sigma\, a_{sc}\, \sigma_y\, L'$$

$$= \Sigma\, a_{sc}\, \sigma_y\, 2\pi \cos\theta \sin^2\theta$$

$$= 2\pi\, a_{sc}\, \sigma_y\, \delta\, \Sigma \cos\theta \sin^2\theta \qquad (3.19)$$

where a_{sc} is the area of cross-section of each circumferential reinforcement bar.

Now the work equation can be written as

$$W = E = E_1 + E_2$$

Substituting the values of W, E_1 and E_2

$$\frac{4}{3}\pi R^2 \delta p = \frac{1}{2} n_m a_{sm} \sigma_y \pi\delta + 2\pi a_{sc} \sigma_y \delta \Sigma \cos\theta \sin^2\theta$$

$$\text{Or,}\ \frac{4}{3} R^2 p = \frac{1}{2} n_m a_{sm} \sigma_y + 2 a_{sc} \sigma_y \Sigma \cos\theta \sin^2\theta \tag{3.20}$$

This equation gives the upper bound to the collapse pressure (p).

3.4.3 Computations

From the structural data,

$n_m = 2 \times 880 = 1760$

$a_{sm} = a_{sc} = 1017.89$

$\sigma_y = 345$ MPa

$R = 20.6$ m

and $\Sigma \sin^2\theta \cos\theta$ for all circumferential reinforcement bars $= 116.8$.

Substituting these values in equation (3.20)

$$\frac{4}{3} \times 424.36\, p = \frac{1}{2} \times 2 \times 880 \times \frac{1017.89}{10^6} \times 345 + \frac{2 \times 1017.89}{10^6} \times 345 \times 116.8$$

Or, $p = 0.84$ MPa

($= 3 \times$ Design pressure)

3.5 Collapse Pressure

Since the cylindrical shell and hemispherical dome are part of the containment structure, failure of any one of these would mean failure of the containment structure. According to the analysis performed in earlier sections, upper bound to the collapse pressure for the dome is less than the

collapse pressure for the cylindrical shell. Hence upper bound to collapse pressure for the containment structure

= Upper bound to the collapse pressure for the dome

= 0.84 MPa

= 3 x Design pressure.

CHAPTER - 4

ANALYSIS FOR ASYMMETRIC PRESSURE AND THERMAL STRESSES

4.0 General

This chapter deals with the analysis of the containment structure subjected to asymmetric internal pressure and temperature loads resulting from LOCA. Since coolant pipes are contained only in the cylindrical portion, any pipe rupture would, generally, lead to short-duration asymmetric pressure and high temperature immediately after the accidents, in the cylindrical portion only. In about 0.5 seconds [4] the pressure and temperature would become uniform. Thus analysis for asymmetric pr2ssure is needed only for cylindrical shell portion while thermal stress analysis should be done for the cylinder as well as the dome. This chapter contains the study of:

(a) Stresses caused by asymmetric pressure in the cylindrical shell.

(b) Thermal stresses in the cylindrical shell.

(c) Thermal stresses in the dome, and

(d) Combined effect of two types of loads in the cylindrical shell.

Since the material properties such as tensile strength and modulus of elasticity do not change much in the temperature range likely in case of LOCA [13], the original (unheated) values of these properties have been used in this chapter.

4.1 Spatial Distribution of Pressure

The exact spatial distributtion of pressure immediately after a LOCA is not known. A cosine variation has been assumed, for this analysis, which has been found to model blast like situations fairly well. The pressure, thus, is given by

$$
\begin{aligned}
p_r &= -p_o \cos\frac{\theta}{2} \cos\frac{\pi z}{R} && \text{for } |z| \leq \frac{R}{2} \\
&= 0 && \text{for } |z| > \frac{R}{2}
\end{aligned}
\qquad (4.1)
$$

where p_o = 0.56 MPa (= 2x design pressure) and coordinate system is as shown in Fig.4(a) and Fig. 4.1(b)

4.2 Analysis of the Cylindrical Portion Subjected to Asymmetric Pressure

This analysis, performed in next few pages, has an element of approximation - essentially because only membrane action has been taken into account. Or in other words, the shell has been treated as a thin-shell. This idealization is justified as diameter to thickness ratio of the shell (=30.5) is considerably more than the critical value (=15) generally accepted.

Consider an element of shell as shown in Fig. 4.2. Stresses acting on this element resulting from membrane action can be treated in form of 'Stress Resultants', which are defined as the total forces acting per unit length of middle surface as given by following expressions [17]:

$$N_\theta = N'_{\theta\theta} = -p_r R \qquad (4.2)$$

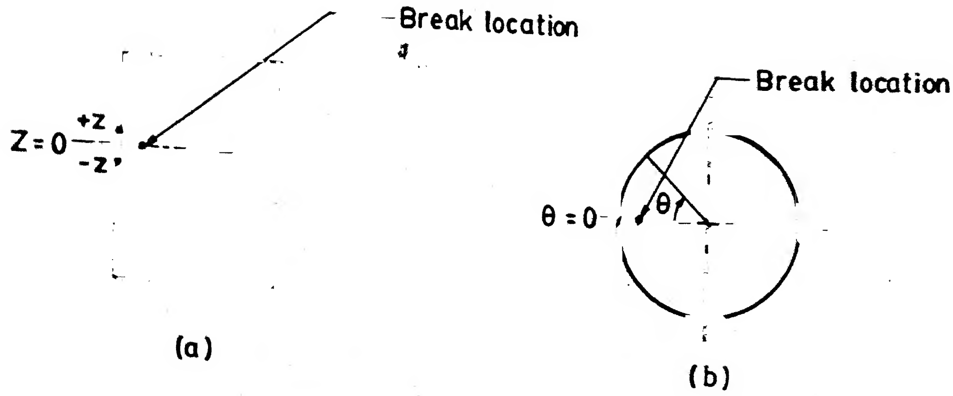

FIG. 4.1 COORDINATE SYSTEM USED IN PRESSURE DISTRIBUTION

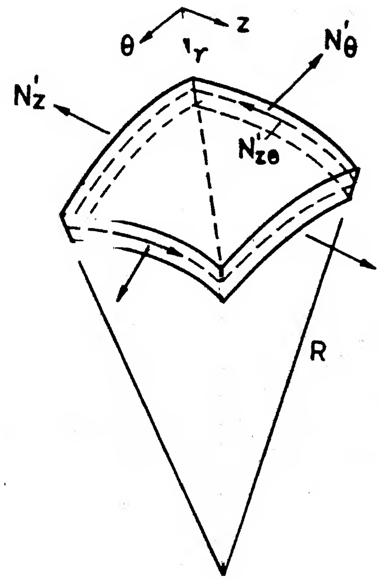

FIG. 4.2 ELEMENT OF THIN CYLINDRICAL SHELL

$$N'_{z\theta} = -\int \left(\frac{1}{R} \frac{\partial N'_\theta}{\partial \theta} + p_\theta \right) dz \tag{4.3}$$

$$N'_z = N'_{zz} = -\int \left(\frac{1}{R} \frac{\partial N'_{z\theta}}{\partial \theta} + p_z\right) dz \tag{4.4}$$

where N'_{ij} is the stress resultant in the i direction on a plane normal to j plane (called j face). Here r, θ and z refer to the radial, circumferential and longitudinal directions respectively. R is the radius of median surface.

Since the shell is subjected to radial pressure only $p_z = p_\theta = 0$.

Internal radial pressure is given by equation (4.1) as

$$p_r = -0.56 \cos\frac{\theta}{2} \cos\frac{\pi z}{R} \quad \text{for } |z| \le \frac{R}{2}$$

$$= 0 \qquad\qquad \text{for } |z| > \frac{R}{2}$$

Since large stresses occur only at section near the break location there is no need to consider the stresses in the region corresponding to $|z| > \frac{R}{2}$.

Substituting the values of pressures in equation (4.2) to (4.4)

$$N'_\theta = 0.56\, R \cos\frac{\theta}{2} \cos\frac{\pi z}{R} \tag{4.5}$$

$$N'_{z\theta} = -\int \frac{1}{R}\, 0.56\, r \cos\frac{\pi z}{2} \left(\frac{1}{2}\right) \left(-\sin\frac{\theta}{2}\right) dz$$

$$= \frac{0.28\, R}{\pi} \sin\frac{\theta}{2} \sin\frac{\pi z}{R} \tag{4.6}$$

$$N'_z = -\int \frac{1}{R} \frac{0.28\, R}{\pi} \sin\frac{\pi z}{R} \frac{1}{2} \cos\frac{\theta}{2}\, dz$$

$$= 0.014\, R \cos\frac{\theta}{2} \cos\frac{\pi z}{R} \tag{4.7}$$

Since the circumferential (or hoop) stress is the critical stress it is appropriate to consider an element which is subjected to maximum circumferential stress.

(a) For maximum N_θ'

$$\frac{\partial N_\theta'}{\partial \theta} = 0 \qquad (4.8)$$

and

$$\frac{\partial N_\theta'}{\partial z} = 0 \qquad (4.9)$$

Equations (4.8) and (4.9) give the following values of θ and z which correspond to maximum value of the circumferential stress resultant

$$\theta = 0$$

$$z = 0$$

Using these value

$$[N_\theta']_{max} = 0.56\ R$$

$$= 11.91 \text{ MPa - m.}$$

Circumferential or hoop stress in steel is given by

$$\sigma_\theta = \frac{\text{Circumferential stress resultant}}{\text{Thickness of equivalent 'steel' shell}}$$

Thickness of Equivalent 'Steel' Shell: For computation of thickness of equivalent 'steel' shell, consider one metre length of the shell.

Transformed area of cross section on θ face in terms of area of steel

$$= \text{Area of steel reinforcement} + \frac{\text{Area of concrete}}{\text{Modular ratio}}$$

i.e. $A_\theta = \left[\frac{4 \times 1000 \times 1017.89}{1^{50}} + \frac{1.35 \times 1 \times 10^6}{13}\right] mm^2$

$= 0.131\ m^2$

Thickness of equivalent 'steel' section,

$$t_\theta = \frac{A_\theta}{\text{Length of shell considered } (= 1\ m)}$$

$$= 0.131\ m$$

Using this values

$$[\sigma_\theta]_{max} = \frac{[N_\theta']_{max}}{0.131}$$

$$= \frac{11.91}{0.131}\ MPa$$

$$= 91\ MPa$$

(b) The corresponding values of shear stress resultant and longitudinal stress resultant:

Substituting values $\theta = z = 0$,

$N_{z\theta}' = 0$

$N_z' = 0.014\ R$

$= 0.30\ MPa - m$

Longitudinal stress $= \dfrac{N_z'}{\text{Thickness of equivalent 'steel' shell}}$

For the computation of thickness of equivalent 'steel' shell, consider 0.175 m long circumferential length of shell. Transformed area of cross section on z face in terms of area of steel.

$$A_z = \text{Area of steel reinforcement} + \frac{\text{Area of concrete}}{\text{Modular ratio}}$$

$$= [(2 + 2.33)\ 706.86 + \frac{236250}{13}]\ mm^2$$

$$= 21233.8\ mm^2$$

$$= 21233.8 \times 10^{-6}\ m^2$$

Thickness of equivalent 'steel' shell

$$t_z = \frac{A_z}{\text{Length considered}} = \frac{21233.8 \times 10^{-6}}{0.175}$$

$$= 0.12\ m$$

Using this value

$$\sigma_z = \frac{N_z'}{0.12}$$

$$= \frac{0.30}{0.12}$$

$$= 2.5\ MPa$$

Stress on a small element around the point $r = R$, $\theta = z = 0$ can be shown in Fig. 4.3.

Since there are no shear stresses acting, normal stresses shown are principle stresses.

Comparing major principal stress (σ_θ) with the yield stress of steel (σ_y)

$$\frac{\sigma_y}{\sigma_\theta} = \frac{345}{91} = 3.79$$

which shows that hoop reinforcement is safe against yielding.

4.3 Thermal Stresses

Subsequent to LOCA temperature inside the containment rises and this leads to a difference in temperature between inside and outside faces of the containment wall. If the two faces were free to expand, inside face will expand more than the outside face. But since the free expansion is prevented, stresses are caused in the section. In this section, thermal stresses in the cylindrical shell and the hemispherical dome have been computed.

4.3.1 Thermal Stresses in Cylindrical Shell

In the cylindrical shell, thermal stresses are generated in two directions: longitudinal and circumferential.

(a) Thermal stresses in longitudinal or vertical direction

The temperature variation across the thickness of the wall can be closely approximated by a linear variation [12].

Consider a vertical section through an element of the cylinderical shell. Let $T^{o}C$ be the temperature difference between inside and outside faces.

Let e be the strain due to temperature difference of $T^{o}C$.

Rise of temperature in tension reinforcement = $(1-a)\ T^{o}\ C$

Free expansion of tension reinforcement = $(1-a)\alpha T$

where α is the coefficient of thermal expansion which is same for steel and concrete.

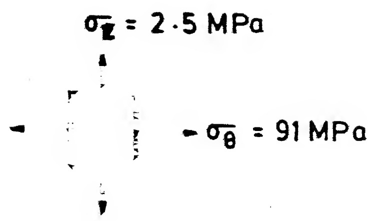

FIG.4.3 STRESSES ON THE CRITICAL ELEMENT

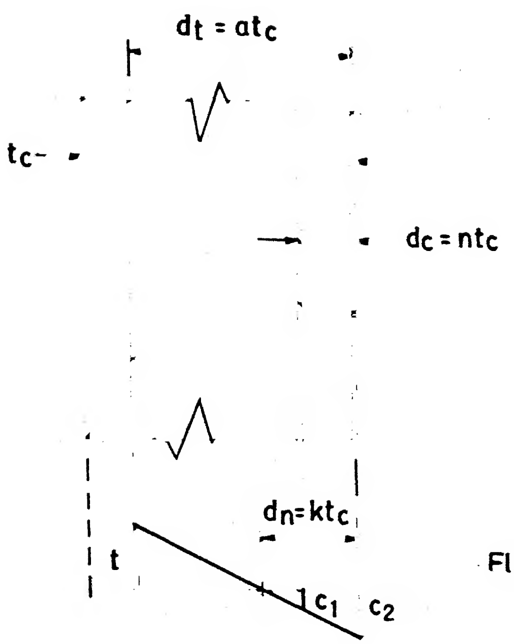

FIG.4.4 THERMAL STRESS DISTRIBUTION IN THE CYLINDRICAL SHELL

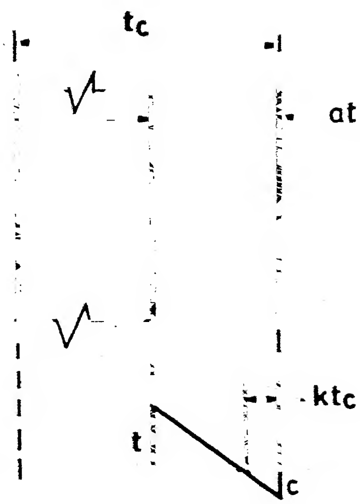

FIG.4.5 THERMAL STRESS DISTRIBUTION IN THE DOME

Tensile stress in tension reinforcement due to difference between strain e and free expansion due to temperature rise,

$$t = E_s [e - (1-a) \alpha T] \quad (4.10)$$

Maximum stress (compressive) on concrete will be on the innermost fibre, where

$$c = E_c (T\alpha - e) \quad (4.11)$$

At the neutral axis, free expansion will be equal to strain e

$$\text{i.e. } e = (1 - k) T\alpha \quad (4.12)$$

Substituting this in (4.10) and (4.11)

$$t = E_s \alpha T (a - k) \quad (4.13)$$

$$c = E_c \alpha T k \quad (4.14)$$

Now, total compressive force = total tensile force

$$\frac{1}{2} c k t_c + c_1 t_{sc} = t_{st} t \quad (4.15)$$

where, area of steel under tension, per unit width $t_{st} = p_t t_e$ and area of steel under compression, per unit width

$$t_{sc} = p_c t_e$$

Here, p_t and p_c are percentage of steel reinforcement under tension and compression respectively.

Substituting these values in Eq. 4.15,

$$\frac{1}{2} c k t_c + c_1 p_c t_e = t p_t t_e \quad (4.16)$$

Applying strain compatibility to the stress distribution shown in Fig. 4.4.

$$t = m\,c\left(\frac{a-k}{k}\right) \tag{4.17}$$

and $$c_1 = m\,c\left(\frac{k-n}{k}\right) \tag{4.18}$$

Substituting the foregoing in equation (4.16) and simplifying gives

$$k^2 + 2m\,(p_c + p_t)\,k - 2m(n\,p_c + a\,p_t) = 0 \tag{4.19}$$

From this equation value of k can be obtained which, then, can be used to calculate t, c and c_1.

(b) Thermal stresses in circumferential direction

Consider unit height of wall. Performing an analysis similar to the one in previous section and equating compressive and tensile force gives the equation

$$k'^2 + 2m\,(p_c' + p_t')\,k' - 2m\,(n'p_c' + a'\,p_t') = 0 \tag{4.20}$$

which gives the position of neutral axis. Here all the symbols are as defined in previous section except for the (') mark which refers to this particular case.

Further, from strain compatibility

$$t' = m\,c'\left(\frac{a'-k'}{k'}\right) \tag{4.21}$$

and $$c_1' = m\,c_1'\left(\frac{k'-n'}{k'}\right) \tag{4.22}$$

Let e be the actual strain.

Stress in concrete $c' = E_c\,(\alpha T + e)$ (4.23)

Stress in tension steel $t' = E_s[e - (1 - a')\alpha T]$ (4.24)

Eliminating e from (4.23) and (4.24)

$$t' + m\,c' = E_s\,\alpha\,T\,a' \tag{4.25}$$

Once k' has been obtained from eq. (4.20), stress in steel and concrete can be computed from (4.21), (4.25) and (4.22). Computation of thermal stresses in the cylindrical shell:

(a) Thermal stresses in the longitudinal direction

Structural data : p_t = 0.68 percent = 0.0068

p_c = 0.62 percent = 0.0062

$$a = \frac{1.15}{1.35} = 0.85$$

$$n = \frac{0.20}{1.35} = 0.15$$

$$m = 13$$

$$E_s = 2\times10^5 \text{ MPa}$$

$$E_c = \frac{E_s}{m}$$

$$\alpha = 1.1 \times 10^{-5}$$

Temperature difference, T = temperature at the inner face

− temperature at the outer face.

$$T \text{ (in } ^\circ\text{C)} = 120 - 20 = 100$$

Substituting these values in equation (4.19)

$$k^2 + 2\times13(0.0068 + 0.0062)\,k - 2\times13(0.0068 \times 0.85 + 0.0062 \times 0.15) = 0$$

or $$k^2 + 0.338\,k - 0.174 = 0$$

or $$k = [-0.338 \pm \sqrt{(0.338)^2 + 4(0.174)}\,]\,/\,2$$

Considering positive root only,

$$k = 0.28$$

$$t = 2\text{x}10^5 \text{ x } 1.1\text{x}10^{-5} \text{ x } 100 \text{ x } (0.85-0.28)$$

$$= 125.4 \text{ MPa}$$

$$c = \frac{2 \text{ x } 10^5}{13} \text{ x } 1.1 \text{ x } 10^{-5} \text{ x } 100 \text{ x } 0.28$$

$$= 4.74 \text{ MPa}$$

$$c_1 = 13 \text{ x } 4.74 \text{ x } \frac{(0.28 - 0.15)}{0.28}$$

$$= 28.61 \text{ MPa}$$

(b) Thermal stresses in circumferential direction

Structural Data:

$$p'_t = 0.0104, \quad p'_c = 0.0096$$

$$a' = 0.85$$

$$n' = 0.15$$

Values of E_c, E_s, m, α and T are same as used in previous section.

Substituting the values in equation (4.20)

$$k'^2 + 2\text{x}13(0.0104 + 0.0096) - 2\text{x}13(0.0104\text{x}0.85 + 0.0096 \text{ x } 0.15) = 0$$

or $\quad k'^2 + 0.52\, k' - 0.26 = 0$

Considering the positive root only

$$k' = 0.40$$

Using equations (4.21) and (4.25)

$$t' = 13\, c' \,(1.125) = 14.325c' \text{ and}$$

$$t' + 13\, c' = 2\text{x}10^5\text{x } 1.1\text{x}10^{-5}\text{x } 100 \text{ x } 0.85 = 187$$

Solving these two equations

$$t' = 99 \text{ MPa}$$

From equation (4.22)

$$c' = 55 \text{ MPa}$$

4.3.2 Thermal stresses in the hemispherical dome

In the hemisphérical dome, thermal stresses are generated in two orthogonal directions : circumferential and meridional. These stresses can be computed using an analysis similar to the one performed for the cylindrical shell. Analysis for stresses in both the directions is essentially the same.

Consider a section through an element of the dome wall as shown in Fig. 4.5.

Equating compressive and tensile forces

$$\frac{c}{2} k\, t_c = A\, t = p\, t_c\, t \tag{4.26}$$

From strain compatibility, $t = mc\left(\frac{a-k}{k}\right)$ (4.27)

Substituting

$$\frac{c}{2} k\, t_c = p\, t_c\, m\, c\left(\frac{a-k}{k}\right)$$

$$\frac{k}{2} = m\, p\left(\frac{a-k}{k}\right)$$

OR
$$k = \sqrt{2\, pma + p^2 m^2} - p\, m \tag{4.28}$$

Let e be the actual strain.

Stress in concrete, $c = E_c\,(\alpha T - e)$

Stress in steel, $t = E_s\,[e - \alpha T\,(1 - a)]$

Eliminating e from these two equations

(4.29)

Stresses can be computed from expressions (4.27) to (4.29).

Computation of Thermal Stresses in Circumferential Direction

Structural data: $a = 0.5$

$p_o = 1.3$ percent $= 0.013$

Material properties E_s, m and have same values as used in the previous section. Substituting these values in equation (4.28)

$$k = \sqrt{2x0.013 \times 13x0.5 + (0.013 \times 13)^2} - 0.013x13$$

$$= 0.28$$

From equations (4.27) and (4.29),

$$t = 13\ c\ (\frac{0.5 - 0.28}{0.28}) = 10.2\ c$$

and $t + 13\ c = 2x10^5 x1.1x10^{-5} x100 \times 0.5 = 110$

From above two equations

$c = 4.74$ MPa

$t = 48.36$ MPa

Computation of Thermal Stress in Meridional Direction

All structural dimensions and values of material properties are same except for percentage of reinforcement. In this case,

$$p = p_m = 0.018$$

Substituting these values in equation (4.28)

$$k = \sqrt{2x0.018x13x0.5 - (0.018x13)^2} - 0.018x13$$

$$= 0.31$$

From equations (4.27) and (4.29)

$$t = 13\ c\ \left(\frac{0.5 \times 0.31}{0.31} \right) = 6.4\ c$$

and $$t + 13\ c = 2 \times 10^{5} \times 1.1 \times 10^{-5} \times 100 \times 0.5 = 110$$

From these two equations

$$c = 5.76\ \text{MPa}$$

$$t = 35.13\ \text{MPa}$$

4.4 Combined Effect in the Cylindrical Shell

Since both the internal pressure and thermal loads cause hoop stress in the tension reinforcement it is important to consider their combined effect. Consider the critical element of section 4.2.

Total circumferential tensile stress

σ_t = Tensile stress due to internal pressure + thermal tensile stress in steel

$$\sigma_t = 91 + 99 = 190\ \text{MPa}$$

Comparing this with yield stress of steel (σ_y),

$$\frac{\sigma_y}{\sigma_t} = \frac{345}{190} = 1.82$$

Hence , failure of hoop reinforcement is not likely.

CHAPTER - 5

SERVICEABILITY CONSIDERATIONS

5.0 General

In this final chapter two more requirements have been considered. These are crack control and damage by internal missile. These considerations may not be of primary importance from the point of view of safety against collapse of structure but they are of great significance when looked upon as serviceability considerations.

Crack, in the containment wall, if wide enough and deep enough, can cause leakage. Though the conventional RCC containments are provided with a steel liner on the inner face of containment to prevent leakage, it may not be sufficient if the cracks are large.

Internal missile in this context refers to any broken part of machinery or pipe that might be thrown away in an explosion. It can have any shape or size depending on the material and magnitude of the explosion. Some standard missile dimensions and properties are used in investigation of damage caused by it. The primary damage caused by the missile is in form of deep penetration which apart from rendering the wall weak, can also help in formation of through-the-wall cracks.

Conclusions drawn from the analysis and limitations of the work have been mentioned in last two sections of this

chapter. Suggestions for further work in this area have also been given.

5.1 Damage by Internal Missile

The most damaging effect of an internal missile is in the form of penetration in the containment wall. This can weaken the wall locally and lead to leakage. The penetrating effect of a missile is generally estimated by some empirical formulae given by different agencies and organizations. The description of critical missile, also, generally varies.

(a) Modified Petry Formula (used by U.S. Navy)[35]

$$D = K A_p \log \left(1 + \frac{V^2}{215{,}000} \right)$$

where D is the depth of penetration in feet

A_p is the sectional pressure (taken to be 408 lb/ft^2)

V is the velocity of missile (taken to be 903 ft/sec)

and K is a factor depending on material ($= 4.76 \times 10^{-3}$ ft^3/lb for 3000 lb/in^2 ($\simeq$ 20 MPa) concrete with approximately 1.4 percent reinforcement).

Using these values

$$D = 4.76 \times 10^{-3} \times 408 \ \mathrm{Log} \left(1 + \frac{(903)^2}{215{,}000}\right)$$

$$= 1.32 \text{ ft}$$

$$= 40 \text{ cm (approximately)}$$

(b) Modified U.S. National Defence Research Committee [37]

$$\frac{D}{d} = \left[\frac{4K\ NW}{d} \left(\frac{V}{1000\ d} \right)^{1.8} \right]^{1/2} \quad \text{for } \frac{D}{d} \leq 2.0$$

where D = depth of penetration in inches

d = missile diameter in inches (taken to be 10 inch)

W = missile weight in pounds (taken to be 68 lb.)

$K = 180/\sqrt{f_c} = 3.3$

N = Missile shape factor (= 1.14 for very sharp nose)

Substituting these values

$$\frac{D}{d} = 1.16$$

Thus D = 11.6 inch

= 30 cm (approximately)

The depth of penetration is much smaller than the wall thickness and hence, not of much concern.

5.2 Crack Criterion

Cracks, in any material, occur when tensile stresses in the material exceed permissible values. In the containment structure tensile stresses are caused by two effects: internal pressure and temperature difference across the containment wall. On the basis of analysis already performed it can be concluded that there is no major threat of cracking in containment wall. This conclusion is essentially based on following considerations:

(a) According to analysis performed in section 4.2 of Chapter 4 the tensile stresses caused due to asymmetric pressure are within permissible limits everywhere. It is appropriate to mention here that stresses caused by uniform

internal pressure would be much lower (as the magnitude of the pressure would be smaller) and would be within permissible limits.

(b) According to results of section 4.3 of Chapter 4, the concrete on the inner face is subjected to compressive stresses everywhere and hence there should not be any cracking on the inner face of the containment wall.

Still, some cracking might take place because of local flaws in the material. But any through-the-wall cracks are not expected. Thus it can be said that crack criterion is satisfied, atleast qualitatively.

5.3 Conclusions

In this thesis an attempt has been made to analyse a typical conventional reinforced concrete containment structure subjected to Loss-Of-Coolant-Accident loads. The plastic analysis has come out with a Load Factor of 3 which implies that structure is reasonably safe against collapse. Analyses for asymmetric pressure and thermal effects have also shown that the containment structure adequately fulfils the safety requirements.

On the basis of discussions in sections 5.1 and 5.2 of this chapter it can be safely concluded that serviceability requirements of crack control and internal missile penetration depth are also satisfied. In short, it can be concluded that the reactor containment, studied in thesis is adequate to withstand LOCA loads.

5.4 Scope for Further Work

Though this work is a step in the desired direction, it has certain limitations. Further work in this field should try to remove them. This can be achieved by a complete analysis by 'Limite State Design' approach. Any such analysis should include quantitative investigation of cracks-including calculation of their width and depth. Dynamic effects of LOCA should be taken into account. Effect of openings in the containment wall can be considered. A reliability based study is also desirable.

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